C5.5 Steel Girders and Beams

See the Office of Bridges and Structures web site for archived Methods Memos listed under articles in this section.

The Methods Memos for which policies have been partially revised and/or for which document references have been updated are noted as partially revised. Any obsolete Methods Memos that apply to this section are listed at the end.

C5.5.2 CWPG LRFD

C5.5.2.1 General

C5.5.2.1.1 Policy overview

Comment: End Span Policy
14 December 2005

The office generally has followed an unwritten policy that the end span should not exceed 54% of the adjacent interior span. Origin of the policy is uncertain but apparently is the result of some study during the design of the first continuous welded plate girder bridge many years ago.

Memo 5.5.2.1.1-2010: Interior/Exterior Girder Design

During design of the I-74 approach spans the engineering consultant performed a detailed comparison of three options: Case 1, separate interior and exterior design sections, Case 2, same section all girders based on Iowa DOT criteria, and Case 3, same section all girders based on Illinois DOT design criteria. Case 1 with the different design sections clearly had an initial cost savings over either of the single section options. Based on this case study it is desirable to consider the option of using different design sections for interior and exterior girders for major bridges.

C5.5.2.1.2 Design information

C5.5.2.1.3 Definitions

C5.5.2.1.4 Abbreviations and notation

C5.5.2.1.5 References

C5.5.2.2 Loads

C5.5.2.2.1 Dead

Methods Memo No. 24: Beam Design and Bearing Design, Distribution of Dead Load 2
4 September 2001

C5.5.2.2.2 Live

Methods Memo No. 182: LRFD Live Load Distribution for Skewed Bridges with Non-standard Rolled Steel Beams, Non-standard Prestressed Beams or Welded Plate Girders
1 July 2008

C5.5.2.2.3 Fatigue

C5.5.2.2.4 Dynamic load allowance

C5.5.2.2.5 Earthquake

C5.5.2.2.6 Construction

~~**Methods Memo No. 183: Policy Regarding Construction Loading**~~
~~**1 January 2008**~~

C5.5.2.3 Load application to superstructure

C5.5.2.3.1 Load modifier

C5.5.2.3.2 Limit states

Memo 5.4.2.3.2, 5.5.2.3.2, and 5.6.2.3.2-2011 ~ Strength V Limit State During Construction and Other Revisions

Based on the description in the AASHTO LRFD Specifications of the Strength V limit state it seemed that it was not intended to be checked during construction. However, a steel plate girder example by M.A. Grubb and R.E. Schmidt distributed nationally by the U.S. Department of Transportation (USDOT) and National Steel Bridge Alliance (NSBA) includes Strength V during construction. The description of Strength V notes: "…plus 1.35 times the design live load (or any temporary live loads acting on the structure when evaluating the construction condition), plus 0.4 times the wind load on the structure, plus 1.0 times the wind on the live load. For evaluating the construction condition under the STRENGTH V load combination, the load factor for temporary dead loads that act on the structure during construction is not to be taken less than 1.25 and the load factor for any non-integral wearing surface and utility loads may be reduced from 1.5 to 1.25." Based on the example and other sources it is clear that Strength V should be checked during construction when appropriate, and articles in the design manual have been revised with respect to construction limit states. (There are several other changes, also.) The steel example is available at the following URL:

http://www.virginiadot.org/business/resources/SteelDesignExample.pdf

C5.5.2.4 Plate girders

C5.5.2.4.1 Analysis and design

C5.5.2.4.1.1 Analysis assumptions

C5.5.2.4.1.2 Materials

Methods Memo No. 78: Charpy Requirements for Steel End Diaphragms
24 July 2003

C5.5.2.4.1.3 Design resistance

C5.5.2.4.1.4 Section properties

C5.5.2.4.1.5 Moment

C5.5.2.4.1.6 Flanges

From 1977 until 2002 the office followed the policy of recommending a flange thickness limit of 2 inches (50 mm). The limit was stated in FHWA Notices N 5040.23 dated 16 February 1977 and N 5040.27 dated 17 August 1977. The office now uses a larger recommended flange thickness limit of 2.5 inches (63 mm) on the basis of Table 4.4 in

Bridge Welding Code, AASHTO/AWS D 1.5M/D1.5: 2002. The minimum preheat and interpass temperature generally is the same for plates 1 ½ to 2 ½ inches (38 to 63 mm) thick.

In the 1970s the office followed a rule that the top flange area should be at least 45% of the bottom flange area. Although no explanation for the rule is available, the rule probably promoted constructibility by ensuring a certain amount of lateral stiffness for a welded plate girder. Because of the constructibility article in the LRFD specifications [AASHTO-LRFD 6.10.3], the office has rescinded the 45% rule and requires that the designer meet the constructibility provisions in the AASHTO LRFD specifications.

Methods Memo No. 103: Plate Thicknesses for Steel Bridges
16 September 2004

Methods Memo No. 131: Continuous Welded Plate Girder Butt-Welded Flange Splice Substitutions
17 August 2006

Partially revised: Methods Memo No. 100: Flange Transitions in Welded Girder Bridges
30 December 2004 (Weight savings was revised on 7 July 2006.)

C5.5.2.4.1.7 Lateral bracing

C5.5.2.4.1.8 Shear connectors

Partially revised: Methods Memo No. 89: Shear Stud Lengths and Haunch Requirements for Steel Girders
26 January 2004(Manual text changed provisions of this memo in 2005 as noted in bold type.)

C5.5.2.4.1.9 Shear

C5.5.2.4.1.10 Web

C5.5.2.4.1.11 Stiffeners

Partially revised: Methods Memo No. 37: Diaphragm Stiffener Connections for Case I
7 January 2002 (The LRFD specifications do not make the Case I and Case II distinction in the standard specifications. As of July 2005 the office still prefers the welded rather than bolted stiffener, however.)

C5.5.2.4.1.12 Deflection and camber

Memo 5.5.4.1.12-2010 CWPG Camber
Large utility pipes were added to the list of dead loads to consider in camber computations.

C5.5.2.4.1.13 Welded connections

C5.5.2.4.1.14 Bolted connections

C5.5.2.4.1.15 Fatigue

C5.5.2.4.1.16 Diaphragms and cross frames

At the time of the January 2006 Bridge Design Manual update the standard cross frames used by the office were redesigned to meet AASHTO LRFD specifications (even though the manual still was based on the AASHTO standard specifications). The following is a summary of the AASHTO and AISC changes that affected the redesign.

- Single angles connected with bolts and welds no longer are permitted to use K = 0.75; K must be 1.0 [AASHTO-LRFD 4.6.2.5]. This 2005 AASHTO LRFD change is in the direction of conservatism and makes published cross frame examples obsolete. Because many cross frame members are at the KL/r<=140 limit for compression members, the change has a significant effect on member size.
- Webs of rolled shapes (and presumably stems of tees) no longer are permitted to be 0.23 inches thick; they now must be 0.25 inches thick [AASHTO-LRFD 6.7.3]. This 1998 or earlier AASHTO LRFD change from the standard specifications is in the direction of conservatism and makes our use of WT 4x9 (with a stem thickness of 0.230 inches) obsolete. We need to use at least a WT 4x10.5.
- Outstanding legs of angles no longer are permitted to have a maximum b/t ratio of 16; they now must have a ratio of 15.89 for A36 steel or less for higher grades of steel or single angles [AASHTO-LRFD 6.9.4.2]. This 1998 or earlier AASHTO LRFD change from the standard specifications is in the direction of conservatism and requires thicker angle legs in some cases.
- For relatively thick angle legs, AISC permits an increase in flexural capacity from $1.25M_y$ to $1.50M_y$. This change in the 2000 AISC single angle specification reduces conservatism in angle capacity for angles with relatively thick legs.

5.5.2.4.1.16-2010 ~ Cross Frame and K-Frame Member Design
Rules added to the 5th Edition of the AASHTO LRFD Specifications in 2010 cover all compression buckling modes and flexure for typical diaphragm and cross frame members, and it is no longer necessary to consult the AISC LRFD Specifications for design of angles and tees.

Where diagonals cross, there is the question of whether the crossing connection can be considered a brace point for out-of-plane buckling. Two papers in AISC's *Engineering Journal* give justification for a brace point if one of the two diagonals is in tension. If both diagonals are in compression, however, the crossing connection is not a brace point for out-of-plane buckling.

Critical cases for design of the cross frames were the following:

- Pier frame: diagonal in completed structure, Strength III for wind, with center brace point
- Intermediate frame: diagonal during construction, Strength III for wind, without center brace point; diagonal during construction, Strength I for deck pour, without center brace point
- Intermediate frame: strut during construction, Strength I for deck pour

C5.5.2.4.1.17 Horizontally curved superstructures

C5.5.2.4.1.18 Additional considerations

Methods Memo No. 65: Limit Steel Girder Lengths Between Field Splices 120 ft
13 May 2002

C5.5.2.4.2 Detailing

Partially revised: Methods Memo No. 151: Steel Bridges Providing Tension and Compression Flange Designation
22 March 2006 (The reference to Article 2408.02, J, 1, is for the 2009 Standard Specifications, revised from 2408.15, A, 2, (c). ~ 16 June 2009)

Methods Memo No. 78: Charpy Requirements for Steel End Diaphragms
24 July 2003

Methods Memo No. 164: Stiffener Clearances
4 September 2007

C5.5.2.4.3 Shop drawings

Methods Memo No. 38: Review of Shop Drawings—Steel Structures
24 January 2002

Obsolete: Methods Memo No. 71: Note on Option of Welding Studs in the Field
18 June 2002

Obsolete: Methods Memo No. 104: LRFD Implementation for Steel Bridge Design
3 June 2005

Obsolete: Methods Memo No. 183: Policy Regarding Construction Loading
1 January 2008